COLD-FORMED STEEL RESEARCH CONSORTIUM

Development of Steel Deck Diaphragm Seismic Design Provisions for ASCE 41/AISC 342

G. Wei, M.R. Eatherton, B.W. Schafer

January 2019

COLD-FORMED STEEL RESEARCH CONSORTIUM
REPORT SERIES
CFSRC R-2019-01

SDII
Steel Diaphragm Innovation Initiative

CFSRC Information

The Cold-Formed Steel Research Consortium (CFSRC) is a multi-institute consortium of university researchers dedicated to providing world-leading research that enables structural engineers and manufacturers to realize the full potential of structures utilizing cold-formed steel. More information can be found at www.cfsrc.org. More information on CFSRC can be found at www.cfsrc.org. All CFSRC reports are hosted permanently by the Johns Hopkins University library in the DSpace collection: https://jscholarship.library.jhu.edu/handle/1774.2/40427.

SDII Information

The Steel Diaphragm Innovation Initiative (SDII) is a multi-year industry-academic partnership to advance the seismic performance of steel floor and roof diaphragms utilized in steel buildings through better understanding of diaphragm-structure interaction, new design approaches, and new three-dimensional modeling tools that provided enhanced capabilities to designers utilizing steel diaphragms in their building systems. SDII was created through collaboration between the American Iron and Steel Institute and the American Institute of Steel Construction with contributions from the Steel Deck Institute, the Metal Building Manufacturers Association, and the Steel Joist Institute in partnership with the Cold-Formed Steel Research Consortium; including, researchers from Johns Hopkins University, Virginia Tech, Northeastern University, and Walter P Moore. More information can be found at www.steeli.org.

National Science Foundation Acknowledgment

This report was prepared with partial support from the U.S. National Science Foundation: NSF-CMMI-1562821: NEESR-CR: Transforming Building Structural Resilience through Innovation in Steel Diaphragms. Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the National Science Foundation.

Development of Steel Deck Diaphragm Seismic Design Provisions for ASCE 41/AISC 342

G. Wei, M.R. Eatherton, B.W. Schafer

1 Abstract

The objective of this report is to provide the background work for the development of recommended seismic design provisions for steel deck diaphragms utilizing ASCE 41 / AISC 342. The current (2017) edition of ASCE 41 for the seismic evaluation and retrofit of existing buildings essentially requires that steel deck diaphragms be designed as elastic elements. This potentially results in large economic and design inefficiencies. Recently existing data has been gathered on the cyclic performance of steel deck diaphragms and this data indicates that appreciable ductility can exist in these systems. Following protocols established in ASCE 41 this document uses existing data to develop acceptance criteria and modeling protocols for seismic performance-based design supported by linear or nonlinear analysis. The method requires fitting a multi-linear model to the cyclic backbone response of available data – and parametrically characterizing the fit to the extent possible. It is found that with minor changes the provisions of AISI S310 may be used to establish strength and stiffness and the available test data to determine ductility and post-peak response. Differences between ductility of diaphragms in buildings and diaphragms in sub-assembly tests are noted and recommendations made to handle this difference. Specific new provisions, ready for adoption by AISC 342 / ASCE 41 are recommended for bare steel deck diaphragms and steel deck diaphragms with concrete fill. A list of future challenges, including the need for additional cyclic testing on steel deck diaphragms with concrete fill, are provided.

2 Motivation

ASCE 41-17 seismic design typically requires steel deck diaphragms to be designed (retrofit or new design) as "force-controlled", i.e. elastic design. In ASCE 7 building design terms "force-controlled" is similar to R=1, or design without force reductions. Such a design philosophy can result in inefficient/uneconomical retrofits. It also implies that a new steel deck diaphragm design per ASCE 7, evaluated per ASCE 41-17, would likely be flagged as requiring diaphragm retrofit. Existing tests indicate that under many circumstances steel deck diaphragms have inelastic/ductile response. Therefore, to remove disconnects and inefficiencies it is appropriate to reevaluate the ASCE 41 steel deck diaphragm design provisions and develop new provisions.

For completeness, and to document the nature of the ASCE 41-17 diaphragm provisions the following excerpts are provided. Highlighting (italics and color) is not from the original.

> "**Acceptance Criteria for Bare Metal Deck Diaphragms.** *Connections* of bare metal deck diaphragms shall be considered *force controlled*. Connection capacity shall be checked for the ability to transfer the total diaphragm reaction into the steel framing. *Diaphragms that are governed by the capacity of the connections* shall also be considered *force controlled*. Bare metal deck diaphragms not governed by the capacity of the connections shall be considered deformation controlled. The m-factors for shear yielding or plate buckling shall be taken from Table 9-64." ASCE 41-17 Section 9.10.1.4

> "**Acceptance Criteria for Metal Deck Diaphragms with Structural Concrete Topping.** *Connections* of metal deck diaphragms with structural concrete topping shall be considered *force controlled*. Connection capacity shall be checked for the ability to transfer the total diaphragm reaction into the steel framing. *Diaphragms that are governed by the capacity of the connections* shall also be considered *force controlled*. Topped metal deck diaphragms not governed by the capacity of the connections shall be considered deformation controlled. The m-factors for shear yielding shall be taken from Table 9-64. " ASCE 41-17 Section 9.10.2.4

For bare metal deck diaphragms the connections nearly always control the shear capacity[1], thus essentially all bare metal deck diaphragms are force-controlled. For metal deck diaphragms with structural concrete topping ASCE 41 does not distinguish between (a) deck which is topped by unreinforced concrete fill, or concrete fill with only temperature and shrinkage steel, and (b) deck which is topped by concrete fill that includes reinforcing bars specifically included to develop higher shear capacity or act as a chord or collector. For (a) current strength calculations in AISI S310-16 include connector capacity, implying all ASCE 41-17 designs should be force-controlled. For (b) no direct guidance currently exists in AISI S310 and AISC 341-16 defers to ACI 318 leaving it unclear as to whether or not such diaphragms must be force controlled or may be designed as deformation controlled.

3 Participants and Process

In 2017 AISC committed to developing a standalone document: AISC 342, that would replace ASCE 41-17 Chapter 9 for structural steel systems. This updating process provided an opportunity to develop and evaluate new steel deck diaphragm design provisions. AISC assigned the development of the new provisions to Task Committee (TC) 7 under the chairmanship of Jim Fisher. A Task Group (TG) was formed in TC7 to develop the new deck diaphragm provisions. The TG was led by Bonnie Manley (AISI) and consisted of current ASCE 41 Chair and AISC TC7 member Bob Pekelnicky (Degenkolb), Tom Sputo (SDI), Jim Fisher (Consultant), and Ben Schafer (JHU). The TG met regularly during 2018 and provided guidance on the provisions as they were

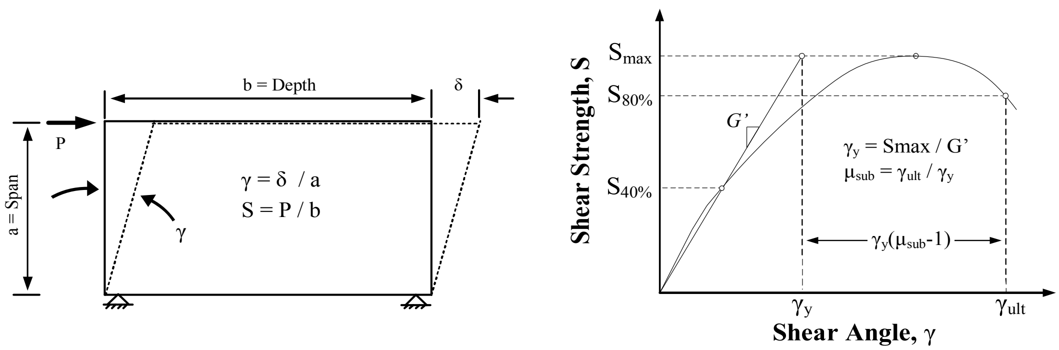
---

[1] This dominance of the connection capacity in controlling diaphragm designs is true for AISI S310-16, but also for design methods previously used in the industry (e.g. Tri-services method).

developed. The developed provisions were balloted in Fall 2018. This report provides a summary of the research that underpins the TG efforts. The research work was conducted by a subset of the Steel Diaphragm Innovation Initiative (SDII) team consisting of Virginia Tech (VT) Ph.D. student Gengrui Wei, his advisor VT Associate Professor Matt Eatherton, and SDII lead Johns Hopkins Professor Ben Schafer

4 Background

As part of the SDII effort a database on the past performance of steel deck diaphragms was compiled and analyzed (O'Brien et al. 2017). The database provides academic and proprietary test results on steel deck diaphragms. A subset of the data includes cantilever tests with complete load-history information. Such tests were processed per Figure 1, and provide an assessment of overstrength and ductility in these systems.



a.) Subassemblage Test Schematic b.) Load-deformation data

Figure 1. Definition of (a) shear angle (γ) and normalized load (S) in typical cantilever test and (b) stiffness (G'), strength (S_{max}), and subassembly ductility (μ_{sub}) evaluated from test results (source: O'Brien et al. 2017)

For cyclic cantilever tests (monotonic tests have observably lower ductility) of bare steel deck diaphragms the response is summarized in O'Brien et al. (2017) Table 6-2 and reprinted here as Table 1. The results indicate that ductility is available in these subassemblies, and that decks connected with mechanical systems (PAF/Screw) have the highest observed ductility.

Direct comparison between like cantilever deck specimens for PAF vs. arc spot weld structural connectors is limited – but a small subset of monotonic tests as detailed in O'Brien et al. (2017) Section 6.2.1 indicates decks with PAF structural connectors and screwed sidelaps have μ_{sub}=3.33 while similar decks with arc spot weld structural connectors and screwed sidelaps (same pattern as the PAF tests) have μ_{su}=2.47. Mechanical structural connectors are superior, but not the only path to subassembly ductility.

Table 1. Cyclic cantilever diaphragm test results for bare deck from O'Brien et al. (2017).

Reference	Spec. ID	G' kip/in	S_{max} kip/ft	γ_y Rad*1000	γ_{ult} Rad*1000	Subassemblage Ductility, μ_{sub}
PAF/Screw; n[1] = 21						
Martin 2002	28	12.1	0.96	6.62	13.1	1.97
Martin 2002	29	15.3	0.92	5.02	6.54	1.30
Martin 2002	31	65.3	1.81	2.31	10.1	4.37
Martin 2002	33	114	2.40	1.75	9.89	5.66
Martin 2002	34	24.7	1.16	3.90	11.9	3.04
Martin 2002	35	26.5	1.18	3.71	5.90	1.59
Essa et al. 2003	8	16.2	0.85	4.38	13.1	2.98
Essa et al. 2003	18	26.3	1.07	3.39	13.5	4.00
Yang 2003	38	23.1	1.03	3.73	13.1	3.50
Yang 2003	40	10.6	0.88	6.95	15.2	2.19
Beck 2008	S 03	72.3	3.96	4.56	14.6	3.20
Beck 2008	S 04	44.9	3.43	6.37	15.3	2.41
Beck 2008	S 05	46.1	3.48	6.29	14.2	2.26
Beck 2008	S 06	73.5	4.33	4.92	13.6	2.76
Beck 2008	S 07	59.6	2.08	2.90	11.0	3.79
Beck 2008	S 08	45.6	1.93	3.53	5.80	1.64
Beck 2013a	C 01	48.7	4.11	7.03	13.3	1.88
Beck 2013a	C 02	61.6	3.93	5.31	12.9	2.42
Beck 2013a	C 03	57.2	5.77	8.42	20.2	2.40
Beck 2013b	S 02C	58.4	3.47	4.95	12.4	2.50
Beck 2013b	S 03C	49.5	4.09	6.88	14.3	2.08
Average		45.3	2.52	4.90	12.4	2.76
Std. dev.		25.1	1.47	1.71	3.30	1.02
Weld/BP[2]; n[1] = 6						
Martin 2002	20	16.8	0.67	3.33	5.04	1.51
Martin 2002	21	15.2	0.93	5.11	6.51	1.27
Martin 2002	36	14.0	0.67	3.99	5.82	1.46
Essa et al. 2003	2	12.3	0.52	3.50	5.06	1.44
Yang 2003	42	11.2	0.70	5.18	12.2	2.36
Yang 2003	48	4.20	0.48	9.47	10.9	1.15
Average		12.3	0.66	5.10	7.60	1.53
Std. dev.		4.06	0.15	2.08	2.88	0.39
Weld/Screw n[1] = 2						
Essa et al. 2003	14	18.3	0.884	4.02	8.04	2.00
Essa et al. 2003	16	16.0	1.30	6.77	12.6	1.86
Weld/Weld; n[1] = 4						
Martin 2002	23	33.0	2.35	5.94	13.1	2.20
Martin 2002	24	26.7	2.27	7.09	10.0	1.40
Essa et al. 2003	12	14.0	0.71	4.25	11.1	2.62
Essa et al. 2003	13	11.2	0.89	6.58	13.1	2.00
Average		21.2	1.55	5.97	11.8	2.06
Std. dev.		8.94	0.76	1.07	1.35	0.44

[1] *n = number of tests in respective group*
[2] *"BP" for button punch*

In addition to the database of cantilever diaphragm tests the lead author participated in a recent series of tests on the cyclic shear performance of sidelap and structural connectors for deck (Torabian et al. 2017,2018a). Specimens were tested to the AISI S905 standard and included 18 - 22 gauge B deck with sidelap connectors: screwed, top arc seam, or button punch; and structural connectors: PAF, arc spot, or arc seam. The results indicate that on the fastener level welds have remarkable strength, but little to no individual connection ductility. PAFs have the highest ductility, though the hysteretic response is highly pinched. A useful feature of the tests is the ability to better understand the absolute deformation the connections can maintain, and what level of post-peak capacity exists at extremely high levels of deformation for any given connection. See Torabian et al. 2018b for a more expansive summary.

The database assembled by O'Brien et al. (2017) also includes cantilever tests on steel deck with concrete fill. The results are limited and SDII is conducting further tests to fill out this test database at this time (http://steeli.org/?p=129). Nonetheless some interesting observations can be made from the existing data. First and foremost, the absolute shear angles available in these systems are small, but in some realistic cases ductility – as traditionally defined – is available. Ductility, in part due to the very small γ_y is highly variable. Note, many of the specimens which are welded only (no shear studs) use detailing that is not considered practical for construction. Considering only those specimens which fail in diagonal tension cracking (DT) or on the perimeter (P) the average ductility is μ_{sub}=3.33, regardless of whether shear studs are included or not.

Table 2. Cyclic cantilever diaphragm test results for filled deck from O'Brien et al. (2017) note all tests in this table from Porter and Easterling 1988.

Spec. ID	Failure Mod[1]	G' kip/in	S_{max} kip/ft	γ_y Rad*1000	γ_{ult} Rad*1000	Subassemblage Ductility, μ_{sub}
Welded; n[1] = 14						
11	S	1770	6.34	0.30	2.25	7.53
12	DT	1710	12.1	0.59	2.30	3.92
13	DT	2020	16.8	0.69	2.23	3.23
14	S	1840	14.0	0.64	5.64	8.85
15	S/DT	1130	6.84	0.50	2.41	4.78
16	DT	920	8.01	0.73	2.39	3.29
17	S	1600	9.70	0.51	5.61	11.1
18	DT	1580	10.7	0.56	2.27	4.03
19	DT	1820	16.5	0.76	1.06	1.40
20	S/P	1300	6.21	0.40	2.25	5.65
21	S/P	870	8.16	0.78	2.56	3.27
22	DT	1650	10.5	0.53	2.09	3.95
23	S/P	1370	7.09	0.43	5.29	12.3
24	DT	1330	11.2	0.71	2.96	4.20
Average		1490	10.3	0.58	2.95	5.53
Std. dev.		338	3.44	0.14	1.39	3.08
Welds with Headed Shear Studs; n[1] = 6						
25	DT	1730	12.0	0.58	2.26	3.92
26	DT	1590	5.80	0.30	1.35	4.45
27	P	1751	6.07	0.29	1.38	4.76
28	P	1580	7.98	0.42	1.41	3.37
29	DT	1890	9.00	0.40	1.24	3.13
30	P	1530	7.69	0.42	1.37	3.27
Average		1670	8.09	0.40	1.50	3.82
Std. dev		131	2.06	0.09	0.34	0.62

[1]DT = Diagonal tension cracking, P = Perimeter fastener failure, S = Shear transfer mechanism failure

Taken together existing research indicates ductility exists in steel deck diaphragm assemblies. Further, sufficient data exists to provide a more informed cyclic backbone response for these systems and thus supply m-factors and nonlinear modeling parameters for ASCE 41-based design.

5 Categorization of steel deck diaphragms and leveraging existing provisions

Categorization of steel deck diaphragms is hampered by an inconsistent set of terminology. ASCE41-17 provides the following two relevant categories: (i) bare metal deck diaphragms and (ii) metal deck diaphragms with structural concrete topping. AISI S310-16 provides the following (i) shear strength and stiffness of profiled steel diaphragm panels and (ii) shear strength and stiffness of steel deck diaphragms with structural concrete or insulating concrete fills. AISC 341-

16 defines "*Composite slab.* Reinforced concrete slab supported on and bonded to a formed steel deck that acts as a diaphragm to transfer load to and between elements of the seismic force resisting system." and refers to a type of composite slab as "concrete slab on steel deck diaphragms" in defining nominal shear strength. Thus, a primary problem with current provisions in all standards, as well as ASCE 41-17 is a lack of clarity on the potential systems. In addition, AISI S310-16 provides some specific language regarding deck vs. profiled panels.

The primary complicating issue has to do with the role of concrete fill on the deck and any reinforcement that is included. Is the fill structural concrete? Is the concrete reinforced? Is the concrete reinforced beyond temperature and shrinkage steel? Is the structural fill reinforced with something other than reinforcing bars (e.g. fibers)? Is the concrete intended to be composite with the steel deck, and if so to what degree? Given the large variety of possibilities, and the need to map these possibilities to existing design provisions where possible, the mapping/categorization is bound to be imperfect. With this proviso the following three categories are ultimately recommended for use in AISC 342/ASCE 41 going forward: (1) Bare Steel Deck Diaphragms, (2) Steel Deck Diaphragms with Reinforced Structural Concrete Topping, and (3) Steel Deck Diaphragms with Unreinforced Structural Topping or Non-Structural Topping.

The suggested terms are a compromise between existing ASCE 41-17 terminology and that of other standards. For (1) "strength and stiffness of bare steel deck diaphragms" can readily be aligned with the scope of AISI S310 including its specific definition of steel deck. For (2) "steel deck diaphragms with reinforced structural concrete topping" is an attempt to separate out the case where the concrete fill is both structural and intentionally reinforced to increase shear capacity or to act as a chord or collector on its boundaries. This case is partially covered in AISC 341-16 and ASCE 41-17, largely by referencing ACI shear wall provisions – and is an area of future standardization need that AISI is seeking to address through a new composite construction committee. For (3) "steel deck diaphragms with unreinforced structural topping or non-structural topping" are covered in the scope of AISI S310-16, providing a ready place to point new provisions towards; however, the title is imperfect as temperature and shrinkage steel or welded wire fabric are often included in such designs – so care must be taken in charging out these new sections.

It is also worth noting that within bare steel deck diaphragms there is a large number of potential variations with regard to connectors and profiles. For strength and initial stiffness these possibilities are largely covered by AISI S310-16, but for m-factors and nonlinear modeling parameters, as required in ASCE 41, there is not guidance for every scenario. As a result, separation similar to Table 1 – which addresses the manner of connection for the structural fastener and for the deck inter-connection/sidelap fastener – but does not divide things further by fastener layouts or spacing, deck type, gauge, etc. is all that is anticipated.

6 Multi-linear fit to cyclic cantilever diaphragm testing

Whether determining m-factors or nonlinear modeling parameters the core of ASCE 41 analysis is understanding the (cyclic) backbone response of the components that comprise a building. This response is generically characterized as shown in Figure 2.

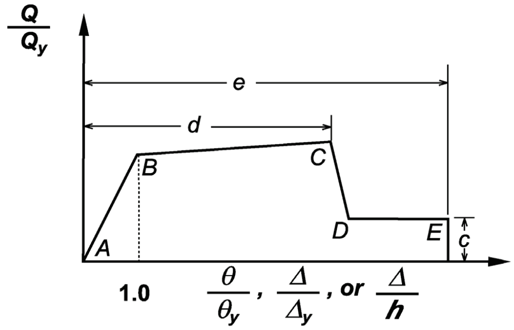
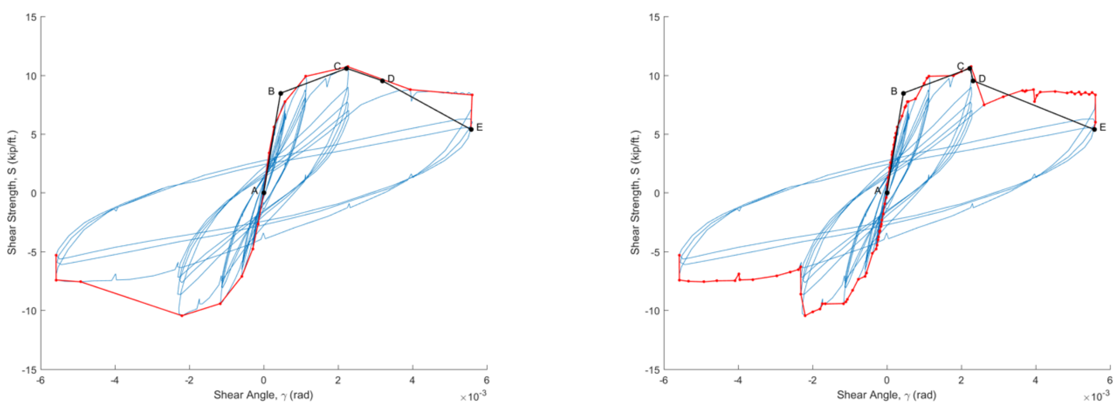


Figure 2. ASCE41 Multi-linear Backbone Response Curve (ASCE 41-17)

Given a measured cyclic response judgment/analysis is still required to convert the nonlinear backbone response to the multi-linear segments of ASCE 41. ASCE 41-17 Section 7.6 provides some guidance in the process as the conversion has a definite impact on the m-factors and nonlinear modeling parameters derived.

Utilizing the data from O'Brien et al. (2017) we first developed a backbone curve from the available cyclic data. As depicted in Figure 3 (red line is the backbone, blue the data) two definitions were initially considered: peak-to-peak and cyclic envelope. The cyclic envelope is closer to the available data, but causes greater complication when fitting the multi-linear segments, and is more dependent on the cyclic loading protocol, particularly in the post-peak characterization. The peak-to-peak model was ultimately selected for the evaluation.

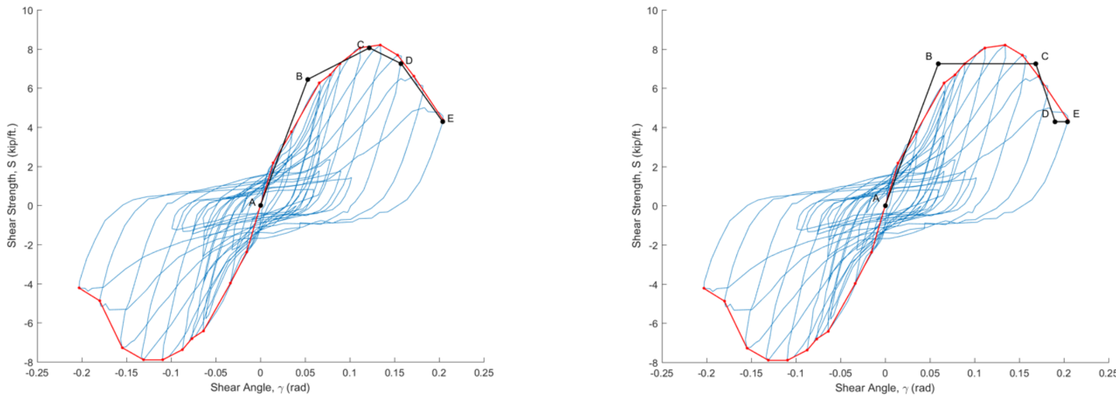


(a) peak-to-peak backbone definition

(b) envelope backbone definition

Figure 3. Backbone definitions (red curves) initially considered in development for example data of a steel deck with concrete fill. Definition (a) peak-to-peak was eventually selected, black segments shown are for initial multi-linear segment model

With the nonlinear backbone established, the ASCE 41 multi-linear segment of Figure 2 must be fit. The initial multi-linear fit set point C at the maximum strength point (in force and displacement) from the test, set B as along the stiffness established at 40% pre-peak but balanced so the energy under ABC is the same as the test data, set point D at the 80% post-peak point (in force and displacement) and set point E as the last point available in the data. An example of this form of multi-linear segment fitting is shown in Figure 4a.



(a) initial multi-linear fit

(b) EEEP multi-linear fit recommended by TG

Figure 4. Comparison of (a) initial bounding fit developed by the team and (b) EEEP fit recommended by TG for bare steel deck diaphragm with PAF structural connector and screwed sidelaps

Based on input from the AISC 7 TG the team also considered and ultimately utilized a different multi-linear segment fit – one more consistent with an Equivalent Energy Elastic Plastic (EEEP) model (but with a degraded post-peak branch). The model parameters are defined in Figure 5 – both pre-peak and post-peak energy is balanced – an example result of this manner of multi-linear fitting is provided in Figure 4b.

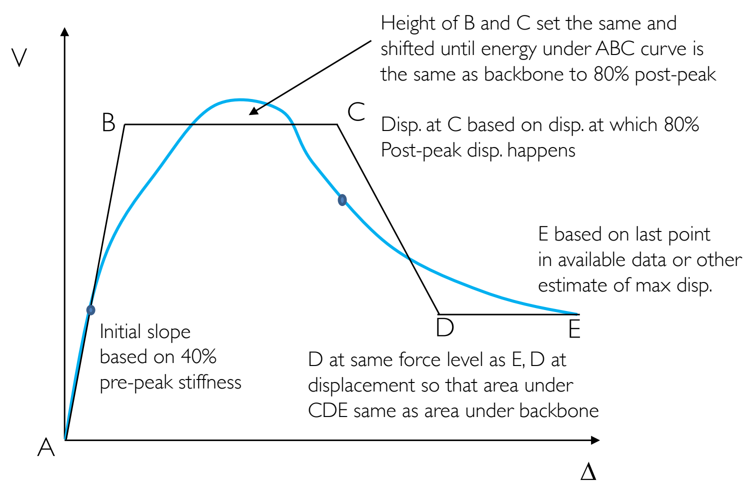
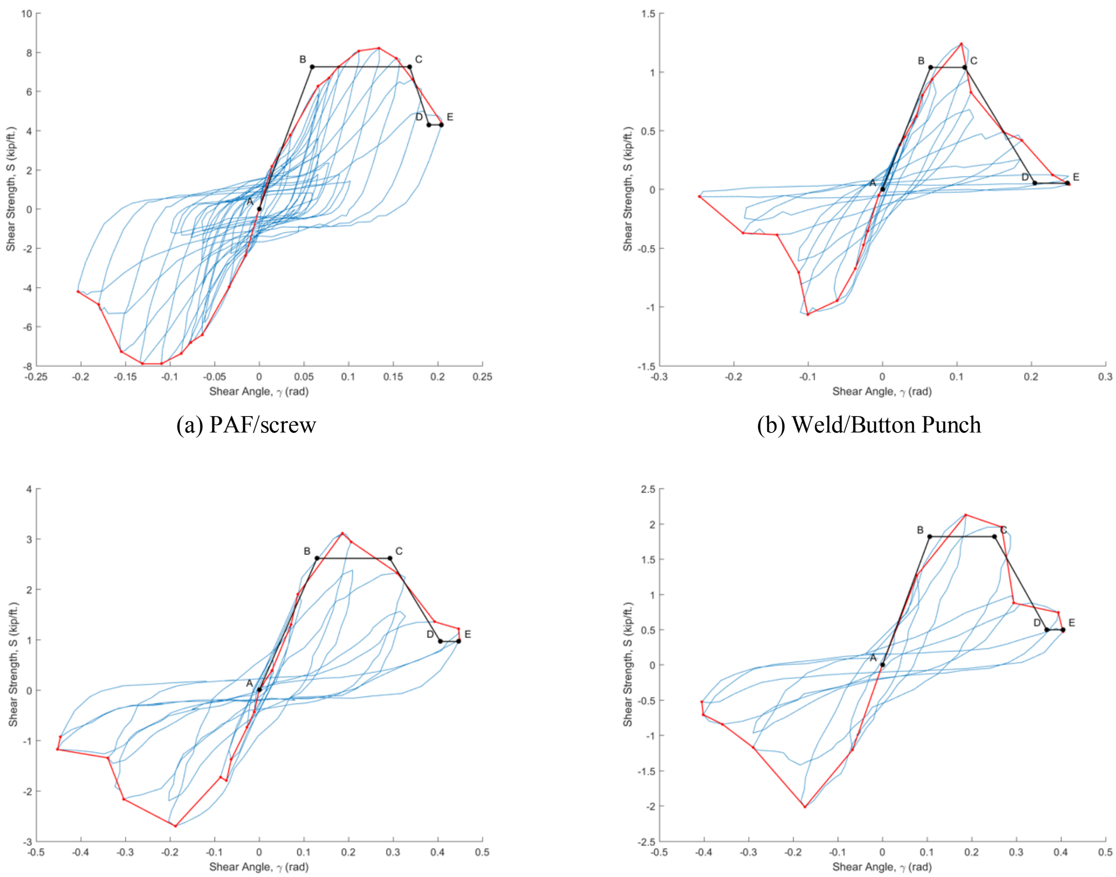


Figure 5. Details of the EEEP model utilized herein

Examples of the ASCE 41 multi-linear segment fit to different bare steel deck diaphragm systems in cantilever diaphragm tests is provided in Figure 6. Some care should be taken in comparing across systems as the axes limits are not equal. Points D and E are sensitive to the amount of post-peak data collected - in some cases the data is quite limited and particularly point E must be used with some level of caution/judgment in subsequent analyses. It is worth noting that backbones are established for the first and third quadrant, but only the first quadrant results are depicted. Ultimately for determining m-factors and nonlinear modeling parameters symmetry in the response is assumed and the values across the 1st and 3rd quadrant are averaged.



(a) PAF/screw (b) Weld/Button Punch

(c) Weld/Screw (d) Weld/Weld

Figure 6. Example EEEP fits to available data for bare steel deck diaphragms

7 Development of subassembly m-factors

Once the multi-linear backbone curves are established, determining the acceptance criteria for linear analysis procedures, i.e. m-factors, is relatively straight forward. ASCE 41-17 Section 7.6 establishes the basic acceptance criteria for immediate occupancy (IO), life safety (LS), and collapse prevention (CP) in linear analysis procedures as a function of the deformation at points B, C, and E. These criteria are summarized in Table 3.

Table 3. Definition of m-factors per ASCE41-17 (reproduced from Ayhan et al. 2016)

	Primary	**Secondary**
$\mathbf{m_{IO}}$	$0.75 \times 0.75 \times 0.67 \times {}^{\Delta_C}/_{\Delta_B}$	$0.75 \times 0.67 \times {}^{\Delta_C}/_{\Delta_B}$
$\mathbf{m_{LS}}$	$0.75 \times 0.75 \times {}^{\Delta_C}/_{\Delta_B}$	$0.75 \times 0.75 \times {}^{\Delta_E}/_{\Delta_B}$
$\mathbf{m_{CP}}$	$0.75 \times min\left({}^{\Delta_C}/_{\Delta_B}, 0.75 \times {}^{\Delta_E}/_{\Delta_B}\right)$	$0.75 \times {}^{\Delta_E}/_{\Delta_B}$

Utilizing the data gathered in O'Brien et al. (2017) we developed the raw estimates for m-factors as provided in Table 4. Given the limitations of the data some judgment is still required in final implementation, but these values provide the basis for selecting the m-factors. As can readily be observed in Table 4 the raw m-factors are greater than 1.0 for life safety and collapse prevention cases (and typically immediate occupancy) indicating the benefit of inelastic response and ductility in these assemblies.

Table 4. Developed subassembly m-factors from existing test data for steel deck diaphragms

(a) primary components

Type	Load	Fastener	Count	m_{IO} avg.	m_{IO} std. dev.	m_{LS} avg.	m_{LS} std. dev.	m_{CP} avg.	m_{CP} std. dev.
Bare	Mono-tonic	PAF / Screw	20	1.5	0.8	2.2	1.1	2.9	1.5
		Weld / BP	8	1.0	0.2	1.4	0.3	1.8	0.5
		Weld / Screw	9	1.2	0.3	1.8	0.4	2.1	0.6
		Weld / Weld	14	1.0	0.3	1.6	0.5	1.8	0.4
	Cyclic	PAF / Screw	21	**1.0**	0.3	**1.5**	0.4	**1.9**	0.5
		Weld / BP	6	**0.7**	0.1	**1.0**	0.1	**1.3**	0.1
		Weld / Screw	2	**1.0**	0.3	**1.6**	0.4	**2.1**	0.5
		Weld / Weld	4	**0.9**	0.3	**1.3**	0.4	**1.6**	0.7
Concrete Filled	Cyclic	Welds only	14	**3.8**	1.4	**5.6**	2.1	**6.5**	2.2
		Welds+studs	6	**2.1**	0.8	**3.2**	1.1	**4.3**	1.5

(b) secondary components

Type	Load	Fastener	Count	m_{IO} avg.	m_{IO} std. dev.	m_{LS} avg.	m_{LS} std. dev.	m_{CP} avg.	m_{CP} std. dev.
Bare	Mono-tonic	PAF / Screw	20	2.0	1.0	3.2	1.8	4.3	2.4
		Weld / BP	8	1.3	0.3	2.1	0.6	2.8	0.8
		Weld / Screw	9	1.6	0.4	2.4	0.8	3.2	1.1
		Weld / Weld	14	1.4	0.4	2.0	0.6	2.7	0.8
	Cyclic	PAF / Screw	21	**1.3**	0.3	**2.1**	0.7	**2.8**	1.0
		Weld / BP	6	**0.9**	0.1	**1.8**	0.4	**2.4**	0.5
		Weld / Screw	2	**1.4**	0.4	**2.7**	1.1	**3.6**	1.5
		Weld / Weld	4	**1.2**	0.4	**2.0**	1.2	**2.7**	1.6
Concrete Filled	Cyclic	Welds only	14	**5.0**	1.9	**7.2**	2.3	**9.6**	3.0
		Welds+studs	6	**2.9**	1.0	**11.1**	2.7	**14.8**	3.6

8 Correction from sub-assembly to full diaphragm

Direct application of the m-factors derived in the previous section implies that the test used to develop the component ductility is under the same conditions and demands as the component in the actual building. For a shear wall, with demands imparted at the floor levels, typical tests impart the same demands as the components. For a diaphragm the situation is more complex. A cantilever diaphragm test imparts uniform shear, as depicted in Figure 7a, while an actual diaphragm sees a varying shear demand across its width – as depicted in Figure 7b. Deformations of the diaphragm, e.g. at midspan, are less than those developed from the uniform shear case that is tested – and these reduced deformations imply a reduced ductility for the diaphragm as a system – and thus reduced m-factors.

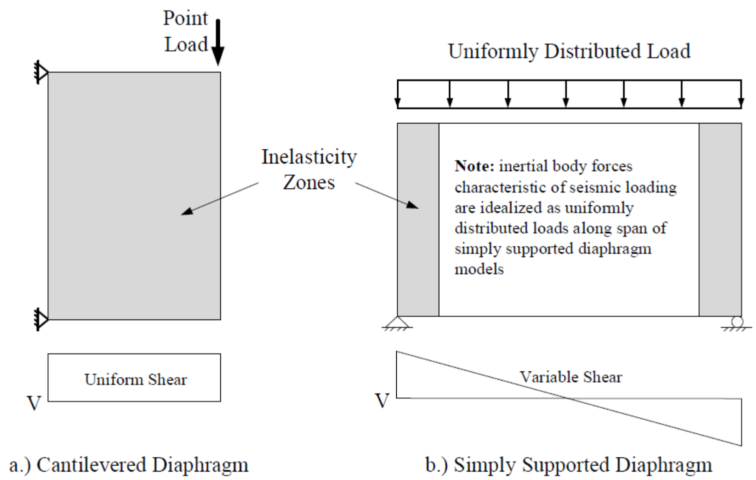


Figure 7. Shear distribution in (a) sub-assembly test and (b) prototypical diaphragm span in a building

O'Brien et al. (2017) developed a correction between sub-assembly cantilever diaphragm tests and simply supported diaphragms. The deformation at midspan was approximated by the summation of an inelastic end zone over diaphragm length L_p and elastic shear deformations over length L. The result of the derivation is that deformation based ductility of the system may be expressed as:

$$\mu_{system} = 1 + 4(\mu_{sub} - 1)\frac{L_p}{L} \qquad \text{(Eq. 1)}$$

Where μ_{sub} is the ductility in the cantilever diaphragm test as reported in Table 1 and Table 2. ASCE 41 m-factors similarly use ratios of deformation to establish ductility therefore the correction for m-factors is the same:

$$m_{system} = 1 + 4(m_{sub} - 1)\frac{L_p}{L} \qquad \text{(Eq. 2)}$$

There is limited information on the width of the plastic zone in actual diaphragms. O'Brien et al. (2017) provides preliminary examinations on L_p/L as does Schafer et al. (2018). At this time a reasonable lowerbound estimate for bare deck diaphragms is $L_p/L \sim 0.1$. This results in system level m-factors as:

$$m_{system} = 0.6 + 0.4m_{sub} \qquad \text{(Eq. 3)}$$

for typical subsystem m-factors (m_{sub}), Table 5 provides the reduced m-factors that would be applicable at the system level.

Table 5. Example system level m-factors

m_{sub}	m_{system}
1.0	1.0
2.0	1.4
3.0	1.8
4.0	2.2

9 Development of nonlinear modeling parameters and acceptance criteria

ASCE 41 supports direct use of component backbone curves (Figure 2) in nonlinear static (pushover) analysis. When such analysis is performed ASCE 41 provides the nonlinear modeling parameters that define the backbone instead of the m-factors. Typically, this in the form of parameters, *d*, *e*, and *c* as illustrated in Figure 8. Development of the backbone curve also requires definition of the initial stiffness and the maximum strength. These are defined for each diaphragm type in relation to AISI S310-16. The initial diaphragm stiffness is selected as G' from AISI S310-16 and the diaphragm strength is the nominal strength S_n from AISI S310-16. R_Ω of Figure 8 refers to correction of the nominal strength to the EEEP strength as derived in Section 6.

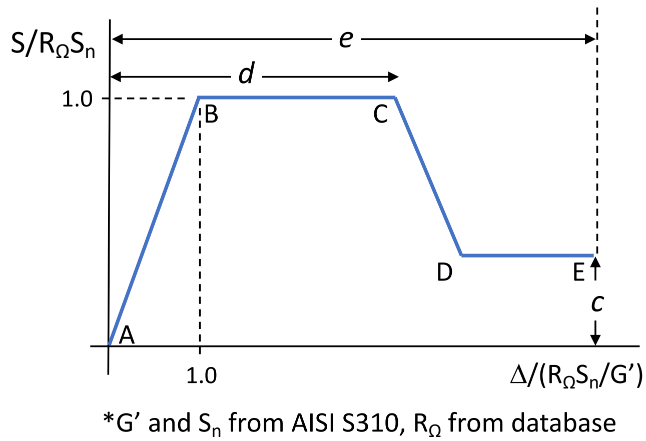


Figure 8. Normalized backbone curve for steel deck diaphragm
(note alternative plastic rotation parameters a and b sometimes used in place of d and e, a=d-1 and b=e-1)

Table 6. Developed nonlinear modeling parameters and acceptance criteria from existing test data for steel deck diaphragms

(a) nonlinear modeling parameters

Type	Load	Fastener	Count	*d* avg	*d* std. dev.	*e* avg.	*e* std. dev.	c^1 avg.	c^1 std. dev.	R_Ω avg.	R_Ω std. dev.
Bare	Mono-tonic	PAF / Screw	20	4.0	2.0	5.8	3.3	0.6	0.2	1.2	0.3
		Weld / BP	8	2.6	0.5	3.7	1.1	0.5	0.2	1.0	0.2
		Weld / Screw	9	3.2	0.8	4.3	1.4	0.6	0.2	1.0	0.2
		Weld / Weld	14	2.8	0.9	3.6	1.1	0.7	0.2	0.9	0.2
	Cyclic	PAF / Screw	21	**2.7**	0.7	**3.7**	1.3	0.6	0.2	1.2	0.2
		Weld / BP	6	**1.7**	0.2	**3.1**	0.7	0.2	0.2	0.9	0.2
		Weld / Screw	2	**2.8**	0.7	**4.8**	2.0	0.3	0.1	0.9	0.1
		Weld / Weld	4	**2.3**	0.8	**3.6**	2.1	0.3	0.3	0.8	0.4
Concrete Filled	Cyclic	Welds only	14	10.0	3.8	12.8	4.0	0.4	0.1	1.8	0.4
		Welds+studs	6	5.7	2.0	19.8	4.9	0.3	0.0	1.5	0.9

Table 6. (continued)
(b) acceptance criteria

Type	Load	Fastener	Count	IO avg	IO std. dev.	LS avg.	LS std. dev.	CP avg.	CP std. dev.
Bare	Mono-tonic	PAF / Screw	20	2.0	1.0	4.0	2.0	5.8	3.3
		Weld / BP	8	1.3	0.3	2.6	0.5	3.7	1.1
		Weld / Screw	9	1.6	0.4	3.2	0.8	4.3	1.4
		Weld / Weld	14	1.4	0.4	2.8	0.9	3.6	1.1
	Cyclic	PAF / Screw	21	1.3	0.3	2.7	0.7	3.7	1.3
		Weld / BP	6	0.9	0.1	1.7	0.2	3.1	0.7
		Weld / Screw	2	1.4	0.4	2.8	0.7	4.8	2.0
		Weld / Weld	4	1.2	0.4	2.3	0.8	3.6	2.1
Concrete Filled	Cyclic	Welds only	14	5.0	1.9	10.0	3.8	12.8	4.0
		Welds+studs	6	2.9	1.0	5.7	2.0	19.8	4.9

1. residual capacity, *c*, is strongly influenced by last data point available in data. Values of c provided here are not recommended for design without adjustment.

For steel deck diaphragms with reinforced structural concrete topping (i.e. including rebar specifically included to enhance strength or provide chord or collector performance) there is not quality test or design guidance, as discussed in Section 5 herein. Therefore, it is recommended that the ASCE 41 provisions for reinforced structural concrete slabs provided in ASCE 41 Section 10.10.2.3 be utilized for nonlinear modeling and acceptance criteria in this case.

10 Expected strength and lowerbound strength

Depending on the retrofit situation it is possible to envision an engineer wanting to consider a steel deck diaphragm as either deformation-controlled (and contributing to seismic energy dissipation) or force-controlled and protected by other components with greater ductility. As a result, it is appropriate that both deformation-controlled and force-controlled procedures be provided for all steel deck diaphragms. This requires definition of expected strength and lowerbound strength in addition to the m-factors, nonlinear modeling parameters, and acceptance criteria.

For bare steel deck diaphragms the expected strength and lowerbound strength are set as a multiplicative factor on the nominal strength, S_n, established from AISI S310-16. In general, the factor is set such that the expected strength, Q_{CE}, is the EEEP strength established in Section 6 and thus where possible follows the R_Ω calculated in Table 6. Here the lower bound strength, Q_{CL}, is set one standard deviation below the expected strength. Based on these criteria, available data, and engineering judgment the Q_{CE} and Q_{CL} of are recommended for bare steel deck diaphragms.

For steel deck diaphragms with reinforced structural concrete topping there is a lack of quality test or design guidance, as discussed in Section 5. Therefore, it is recommended that the ASCE 41 provisions for reinforced structural concrete slabs provided in ASCE 41 Section 10.10.2.3 be utilized for strength.

For steel deck diaphragms with unreinforced structural topping or non-structural topping the strength provisions of AISI S310 are applicable. Therefore, the corrections for expected and lowerbound strength follow the same basic approach as for bare steel deck diaphragms and are provided in Table 8. As noted in the Table the committee that oversees AISI S310 currently has ballots on its docket to revise the capacity calculation for this case. If passed these strength values (particularly Q_{CE}) would have to be revised in a future edition of AISC 342/ASCE 41.

Table 7. Recommended expected and lowerbound strength for bare steel deck diaphragms

Type	Limit	Detail	Q_{CE}	Reasoning
Bare metal deck	Panel buckling		$1.1S_n$	Based on Table K2.1.1-1 AISI S100-16 and Commentary table in AISI S310-16. Arguably 1.0 could be used since it is a buckling limit state.
	Connector	PAF	$1.2S_n$	Based on full-scale diaphragm tests and fit per Table 6.
		Weld	$1.0S_n$	Table 6 places this factor at ~0.9, test-based calibration criteria in AISI S310 would use at least 1.1 – engineering judgment to use $1.0S_n$ here.
		Screw	$1.0S_n$	
		Other	$1.0S_n$	
			Q_{CL}	
Bare metal deck	Panel buckling		$0.9S_n$	Based on Table K2.1.1-1 AISI S100-16 and Commentary in AISI S310-16. Arguably 1.0 could be used if 1.1 is used for Q_{CE}.
	Connector	PAF	$1.0S_n$	Q_{CE}-$0.2S_n$ based on full-scale diaphragm test data.
		Weld	$0.8S_n$	Table 6 data places this between $0.6S_n$ and $0.9S_n$ – small sample size Q_{CE}-$0.2S_n$ selected based on engineering judgment.
		Screw	$0.9S_n$	Q_{CE}-$0.1S_n$ for other fasteners based on test data and judgment.
		Other	$0.9S_n$	

Table 8. Recommended expected and lowerbound strength for "unreinforced" filled steel deck diaphragms

Type	Detail	Q_{CE}	Reasoning
Filled[1] metal deck	Weld	$1.8S_n$	Based on full-scale diaphragm tests and fit per Table 6.[2]
	Studs	$1.5S_n$	Based on full-scale diaphragm tests and fit per Table 6.[2]
		Q_{CL}	
Filled[1] metal deck	Weld	$1.0S_n$	Data is limited, based on Table 6 Q_{CL} should be $0.4S_n$ to $0.9S_n$ lower than Q_{CE}. Given lack of clarity use of $1.0S_n$ recommended as a rational lowerbound for now.
	Studs	$1.0S_n$	

1. structural concrete fill with temperature and shrinkage steel or WWF only
2. Current test-to-predicted ratios control high values of QCE, these need to be revisited in the future if AISI S310 adopts new strength provisions.

11 Adjustments and recommendations for design

Ultimately neither the m-factors nor the nonlinear modeling parameters developed may be used directly for design. Judgment is required to incorporate the fact that the sample sizes are relatively small, that the data is limited particularly for large post-peak excursions, and that the new provisions must work and be aligned with existing related provisions. In addition, discussion is also required as to whether or not the system reductions of Section 8 are to be incorporated. The AISC TG7 task group took all of this under consideration and B.W. Schafer as ballot champion prepared provisions for AISC 342 Ballot 1. The key elements of the developed provisions are provided here with additional explanations.

For bare steel deck diaphragms key recommended provisions are summarized in Table 9. Stiffness (G') is per AISI S310-16. Strength (Q_{CE} or Q_{CL}) is per the recommendations in Table 7. The m-factors are similar to Table 4; however, IO values have been increased to 1.0 per ASCE 41 provisions, LS and CP values have had significant digits reduced wherever possible. Note, the system level m-factor reductions of Section 8 have not been applied. This is a matter of some contest amongst the TG members – the research team stands by the validity of their findings, but the TG notes that this reduction is not applied for other diaphragm systems. At this time, we have decided to recommend using the subsystem m values, but ASCE 41 should consider this issue for future work across all diaphragm systems.

For bare steel deck diaphragms the recommended nonlinear modeling parameters and related acceptance criteria are provided in the second half of Table 9. The data is normalized to the shear angle at yield $\gamma_y=Q_{CE}/G'$. Where possible the results of Table 6 are directly employed. For residual strength parameter, c, the data is limited and overly optimistic. The residual strength values were reduced to 0.4 for PAF/screw bare steel deck and 0.05 for welded steel deck. The PAF/screw residual strength is based in part on the residual strength of a single PAF in shear (Torabian et al. 2017). The weld residual strength is also (conservatively) based on individual strength of welds – and is intended to recognize that little residual strength may exist in these connections, particularly at CP level limits. For shear strength limited by panel buckling (a rare occurrence) the nonlinear modeling parameters are aligned with the recommended m-factors. The residual strength, c, is approximated as being between the PAF/screw case (c=0.4) and a Steel Plate Shear Wall (c=0.7 in ASCE 41-17). A value of c=0.5 is recommended in Table 9.

For steel deck diaphragms with unreinforced structural topping or nonstructural topping key recommended provisions are summarized in Table 10. Stiffness (G') is per AISI S310-16. Strength (Q_{CE} or Q_{CL}) is per the recommendations in Table 8. The recommended m-factors are rounded down from Table 4 and aligned with ASCE 41-17 Table 10-21 for reinforced concrete structural walls. Note, the IO m-factors are rounded down significantly to reflect the desire to limit cracking, which is beyond the original determination of Table 4. The recommended nonlinear modeling parameters and related acceptance criteria are provided in the second half of Table 10. The data is normalized to the shear angle $\gamma_i=Q_{CE}/G'$. Note, consistent with Table 2, typical γ_i are only 40-50 millirads. The modeling parameters are rounded from analysis of available data (Table 6) and approximately aligned with ASCE 41-17 Table 10-20 for reinforced concrete walls controlled by shear. Note, the residual strength ratios are relatively large (c=0.3 or 0.4 recommended in Table 10) but this is at shear angles associated with CP which are only $10\gamma_i$ or 400-500 millirads. Analysts are cautioned that at higher shear angles the residual strength will continue to degrade.

For steel deck diaphragms with reinforced structural concrete topping it is recommended for strength to use ASCE 41-17 Section 10.10.2.2 and for stiffness ASCE 41-17 Section 10.10.2.3 for the slab above the top flute of the deck. For linear procedures ASCE 41-17 Table 10-22 are recommended for the acceptance criteria and for nonlinear procedures ASCE 41-17 Table 10-20 for the nonlinear modeling parameters and acceptance criteria.

Table 9. Recommended acceptance criteria and nonlinear modeling parameters for bare steel deck diaphragms

Table 9a
Acceptance Criteria for Linear Procedures—Bare Steel Deck Diaphragm

Component/Action	*m*-factors for Linear Procedures				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Shear strength controlled by connectors:					
support: PAF; side-lap: screw	1.0	1.5	2.0	2.0	3.0
support: weld; side-lap: screw	1.0	1.5	2.0	2.0	3.0
support: weld; side-lap: button punch	1.0	1.0	1.3	2.0	3.0
support: weld; side-lap: weld	1.0	1.3	1.6	2.0	3.0
Shear strength controlled by panel:					
buckling	1.25	2.0	3.0	2.0	3.0

CP = collapse prevention performance level as defined in ASCE/SEI 41 Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41 Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41 Chapter 2

*Note, panel buckling is the same as used previously in Table 9-5 of ASCE 41-17

TABLE 9b
Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—
Bare steel deck diaphragms

Component or Action	Modeling Parameters			Acceptance Criteria		
	Plastic Rotation Angle, rad.		Residual Strength Ratio	Plastic Rotation Angle, rad.		
	d	*e*	*c*	IO	LS	CP
Shear strength controlled by connectors:						
support: PAF; side-lap: screw	$2.7\gamma_y$	$3.7\gamma_y$	0.4	$1.4\gamma_y$	$2.8\gamma_y$	$4.0\gamma_y$
support: weld; side-lap: screw	$2.8\gamma_y$	$4.8\gamma_y$	0.05[b]	$1.4\gamma_y$	$2.8\gamma_y$	$4.0\gamma_y$
support: weld; side-lap: button punch	$1.7\gamma_y$	$3.1\gamma_y$	0.05[b]	$0.9\gamma_y$	$1.7\gamma_y$	$3.1\gamma_y$
support: weld; side-lap: weld	$2.3\gamma_y$	$3.6\gamma_y$	0.05[b]	$1.2\gamma_y$	$2.3\gamma_y$	$3.6\gamma_y$
Shear strength controlled by panel:						
buckling	$3.6\gamma_y$	$5.6\gamma_y$	0.5	$1.8\gamma_y$	$3.7\gamma_y$	$6.0\gamma_y$

CP = collapse prevention performance level as defined in ASCE/SEI 41 Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41 Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41 Chapter 2

[a] Values are for shear walls with stiffeners to prevent shear buckling.
[b] Structural connectors generally control residual strength. Value based on arc spot weld, arc seam weld c=0.15

Table 10. Recommended acceptance criteria and nonlinear modeling parameters for steel deck diaphragms with unreinforced structural topping or nonstructural topping

Table 10a
Acceptance Criteria for Linear Procedures—Steel Deck Diaphragm with Unreinforced Structural Topping or Nonstructural Topping

Component/Action	*m*-factors for Linear Procedures[a]				
		Primary		Secondary	
	IO	LS	CP	LS	CP
Shear strength of deck with nonstructural topping					
deck welded to support (arc spot or arc seam)	1.5	4.0	6.0	6.0	8.0
headed shear studs welded through deck to support	1.5	3.0	4.0	6.0	8.0

CP = collapse prevention performance level as defined in ASCE/SEI 41 Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41 Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41 Chapter 2

[a] Regardless of the modifiers applied, *m* need not be taken less than 1.0.

TABLE 10b
Modeling Parameters and Acceptance Criteria for Nonlinear Procedures—
Steel Deck Diaphragm with Unreinforced Structural Topping or Nonstructural Topping

Component or Action	Modeling Parameters			Acceptance Criteria		
	Plastic Rotation Angle, rad.		Residual Strength Ratio	Plastic Rotation Angle, rad.		
	d	*e*	*c*	IO	LS	CP
Shear strength of deck with nonstructural topping						
deck welded to support (arc spot or arc seam)	$8.0\gamma_i$	$10.0\gamma_i$	0.4	$2.0\gamma_i$	$8.0\gamma_i$	$10.0\gamma_i$
headed shear studs welded through deck to support	$8.0\gamma_i$	$10.0\gamma_i$	0.3	$2.0\gamma_i$	$8.0\gamma_i$	$10.0\gamma_i$

CP = collapse prevention performance level as defined in ASCE/SEI 41 Chapter 2
IO = immediate occupancy performance level as defined in ASCE/SEI 41 Chapter 2
LS = life safety performance level as defined in ASCE/SEI 41 Chapter 2

12 Discussion

Steel deck diaphragms have appreciable ductility and the developed provisions allow engineers utilizing ASCE 41 / AISC 342 to utilize this fact in their designs and retrofits. In the development of these provisions a number of issues for future work emerged that are worthy of documenting.

- ASCE 41 should consider the use of system level reductions (e.g., m_{system}) for diaphragms similar to that detailed in Section 8 of this report.
- Additional criteria defining the specific details (type of PAF, profile and gauge of deck, etc.) for any of the rows in Table 9 and Table 10 may be needed in the long term for more carefully defining applicability.
- Given the large variety of different details in steel deck diaphragm systems modeling methods that can replace the m-factors or even the overall backbone curves should be enabled and pursued (see Schafer et al. 2018 for an example of model-based steel deck diaphragm determination built-up from fastener information).
- Since the strength values of the recommended provisions are largely tied to AISI S310, any changes in AISI S310 need to be propagated to AISC 342. For example, current AISI S310 ballots may modify the nominal strength of steel deck diaphragms with unreinforced structural topping – this change would need to be included in future additions of AISC 342.
- Additional cyclic test data is needed on concrete-filled steel deck diaphragms.
- A comprehensive approach is needed to the modeling of steel deck diaphragms with reinforced structural topping. AISC 341, AISI S310, and ASCE 41 all essentially defer to ACI 318 shear wall provisions. Novel thinking and new provisions are needed for this case.
- Comparisons of new diaphragm designs by ASCE 7 and ASCE 41 would be instructive.
- Application of these recommended provisions in archetypical structures or actual retrofit designs would be useful for disseminating their practical impact.

Taken as a whole the new provisions fill a gap in steel systems seismic design, but much work remains to provide optimized solutions.

13 Conclusions

Seismic retrofit and rehabilitation following ASCE 41-17 essentially requires that steel deck diaphragms be designed as elastic elements. Substantial experimental evidence exists that steel deck diaphragms have useful levels of ductility in many configurations. Existing data on steel deck diaphragms was analyzed for the purposes of determining acceptance criteria for linear and nonlinear analyses consistent with the performance-based seismic design levels of ASCE 41. A series of m-factors (ductility measures) and nonlinear modeling parameters (multi-linear cyclic backbone curves) were determined for bare steel deck diaphragms and steel deck diaphragms with structural concrete fill (i.e., with no/incidental reinforcement). The resulting analyses form the basis for new ASCE 41 provisions and are aligned with the strength and stiffness provisions provided by AISI S310 and AISC 341. The provisions are recommended for adoption in the first edition of AISC 342, which is intended to replace ASCE 41 Chapter 9 for structural steel systems.

14 Acknowledgments

This report was prepared with partial support from the Steel Diaphragm Innovation Initiative (SDII). SDII is funded by the American Iron and Steel Institute, the American Institute of Steel Construction, the Steel Deck Institute, the Metal Building Manufacturers Association, and the Steel Joist Institute. This report was prepared with partial support from the U.S. National Science Foundation: NSF-CMMI-1562821: NEESR-CR: Transforming Building Structural Resilience through Innovation in Steel Diaphragms. Any opinions, findings, and conclusions or recommendations expressed in this publication are those of the author(s) and do not necessarily reflect the views of the National Science Foundation or other sponsors.

15 References

AISC 341-16 (2016). "Seismic Provisions for Structural Steel Buildings." AISC.

AISI S310-16 (2016). "North American Standard for the Design of Profiled Steel Diaphragm Panels." AISI

AISI S905-17 (2017). "Test Standard for Determining the Strength and Deformation Characteristics of Cold-Formed Steel Connections." AISI

ASCE 41-17 (2017). "Seismic Evaluation and Retrofit of Existing Buildings." ASCE-SEI.

ASCE 7-16 (2016). "Minimum Design Loads and Associated Criteria for Buildings and Other Structures." ASCE-SEI.

Ayhan, D., Madsen, R.L., Schafer, B.W. (2016). "Progress in the Development of ASCE 41 for Cold-Formed Steel." Proceedings of the 23rd International Specialty Conference on Cold-Formed Steel Structures, Baltimore MD. 417-432.

O'Brien, P., Eatherton, M.R., Easterling, W.S. (2017) "Characterizing the load-deformation behavior of steel deck diaphragms using past test data." CFSRC Report R-2017-02, 243 pp., permanent link: jhir.library.jhu.edu/handle/1774.2/40633.

> Also see: O'Brien, P., Eatherton, M.R., Easterling, W.S., Schafer, B.W., Hajjar, J.F. (2017) "Steel Deck Diaphragm Test Database v1.0." CFSRC Report R-2017-03, permanent link: jhir.library.jhu.edu/handle/1774.2/40634.

Schafer, B.W., Smith, B.H., Torabian, S., Meimand, V., Eatherton, M.R. (2018). "Modeling and Performance of Thin-walled Steel Deck in Roof Diaphragms under Seismic Demands." Proc. of the Int'l. Conf. on Thin-walled Structures. Lisbon, Portugal.

Torabian, S. (2017) "Cyclic performance and characterization of steel deck connections." NBM Technologies report to AISI/SDI/SJI, 14 December 2017

Torabian, S. (2018a) "Button Punch Sidelaps (Cyclic testing program)" NUCOR-Verco/Vulcraft Group, Project number 103-042-18, 11 July 2018 (released by Verco for public use)

Torabian, S., Fratamico, D., Shannahan, K., Schafer, B.W. (2018b). "Cyclic performance and behavior characterization of steel deck sidelap and framing Connections." Proceedings of the Wei-Wen Yu International Specialty Conference: Cold- Formed Steel Structures – 2018, November 7-8, St. Louis, MO.